1-7991256

MRL 87-91 (TR) c.2

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PRELIMINARY GEOMECHANICAL ASSESSMENT OF THE ELDRICH-FLAVEL MINE

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Canadian Mine Technology Laboratory

JULY 1987

MINING RESEARCH LABORATORIES
DIVISIONAL REPORT MRL 87-91(TR)

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by

M.C. Bétournay*

ABSTRACT

This report outlines observations based on a short visit to the Eldrich mine. Although the site has been generally characterized as a result of this tour, more detailed investigation is required, especially since no relevant data exists on soil, rock mass or ground behaviour with mining.

The small open stopes in the western portion of the mine will be expanded to surface crown pillar dimensions. However, most of the mining activity will take place in a large zone east of these. There, five or six consecutive open stopes will be placed on-strike of this tabular, 38° dipping gold orebody. Each stope will span 24 m; 6 m rib pillars will be placed between the stopes.

Although there are four recognizable joint families and a poorly developed schistocity in and around the orebody, major stability problems will be incurred because of the presence of a gouge-filled fault, with surrounding weak schist zones. This fault is expected to intersect all the rib pillars in the upper part of the large zone. Preliminary pillar stress calculations show that these pillars may be too weak in the lower area of the large zone. Roof stability will also be affected under the large extraction ratio planned.

A comprehensive study to establish the stability of underground openings under various mining strategies is given.

KEYWORDS: rock mass, rib pillars, surface crown pillars, pillar strength, rock mass characterization, joint survey, fault, fault gauge, open stopes, pre-shear blasting, numerical modelling, monitoring.

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ÉVALUATION PRÉLIMINAIRE GÉOMÉCANIQUE
DE LA MINE ELDRICH-FLAVEL

par
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RÉSUMÉ

Ce rapport décrit les observations faites après une courte visite à la mine Eldrich. Bien que le site a été caractérisé sommairement suite à cet inspection, de plus amples investigations sont requises puisqu'il n'existe pas d'information appropriés sur le comportement du mort-terrain massif rocheux ont des terrains suite à l'extraction minière.

Les petits chantiers ouverts du secteur ouest de la mine seront agrandis, laissant des piliers de surface. Cependant, la moyenne partie de l'activité minière se fera dans une zone majeure située à l'est de ceux-ci. A cet endroit, cinq ou six chantiers consécutifs seront placés sur l'axe de l'azimuth de ce gisement d'or tabulaire à pendage de 38°. Chaque chantier sera 24 m de large; des piliers de paroi de 6 m sépareront les chantiers.

Bien qu'il existe quatre farvelle de diaclases et une schistocité mal développée dans et adjoignant le dépôt, des problèmes majeurs de stabilité existeront à cause d'une faille remplie de boue et accompagnée de zones schistoses. Cette faille intersectera tous les piliers au niveau supérieur de la zone majeure. Des calculs préliminaires de contraintes imposées aux piliers situé au niveaux inférieurs de la zone majeure indiquent que ces piliers peuvent être trop faible. La stabilité des toits sera également affectée par le haut pourcentage d'extraction planifié.

Une étude compréhensive pour établir la stabilité d'ouvertures souterrains est suggérée.

MOTS CLÉS: massif rocheux, piliers de paroi, piliers de surface, résistance de pilier, caractérisation de massif rocheux, sondage de diaclase, faille, boue de faille, chantiers ouverts, sautage périmétrique, modélisation numérique, surveillance.

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1. INTRODUCTION

In July 1987, CANMET performed a preliminary geomechanical assessment of the Eldrich-Flavel Mine, Evain, Quebec. This brief site visit was requested by Mines Sullivan Inc. as a first step in a projected stability research to formulate an evaluation of mine stability under existing site conditions and planned mining strategy. The purpose of this report is to describe, in a summary fashion, the existing site conditions and, by way of pertinent discussion and reflexion, arrive at a summary of items required for a complete program to establish the stability of future mining activity.

2. BACKGROUND

The Eldrich-Flavel gold mine is located in Quebec, 20km north-west of Rouyn-Noranda, near Evain, figure 1. The site is situated in the Superior Province of the Canadian Shield.

The host rocks are tonalite and diorite, portions of the Flavian Pluton. It is 17 x 8 km in extent, the largest of regional intrusives. The diorite, the youngest rock type, cuts older ones such as tonalite. The mineralization consists of a long series of small lenzes, striking NE-SW and dipping ~38° SE, figure 2-3. The gold is finely disseminated, found in the tonalite country rock and diorite intrusive and believed to have been concentrated by the action of, and along, a wrench fault (1). The ore grades 6.4g Au/T and 1.03g Ag/T.

Between 1955 and 1962, Eldrich Mines Ltd. carried out underground mining to a depth of 300 m, creating several long drifts and open stopes which remained unsupported. In 1984, Mines Sullivan Inc. acquired the mining rights. Subsequently, diamond drilling activity revealed a more extensive zone situated east of the lenzes.

The extraction plan is to mine the few remaining small lenzes (~60 m wide, 2-6 m thick) , expanding some until surface crown pillars are created. The bulk of the mining activity, however, will be in the new lenz (zone 5, ~120 m wide, 2-6 m thick). Fill is considered uneconomical, therefore bolting and pillars in zone 5 and bolting of the west stopes are the only means of support considered. Stoping of the west lenzes has already begun; creation of the surface crown pillars will begin in mid 1988, zone 5 will begin production in March 1988. The mine life will be about 5 years.

No geomechanical studies or assessment has been performed at Eldrich. The mine has not conducted surveys related to soil or rock mass conditions. No relevant data exists to complement stability studies.

3. UNDERGROUND VISIT

The underground tour included inspection of drifts and stopes of the west lenzes from level 2 to 6, and level 6 around zone 5.

Several structural elements are present at Eldrich. These create various stability sectors: hanging wall-orebody contact, fault zone and other mass joint (orientation, extent, spacing, roughness) and block size distribution.

The main discontinuity appearing at the site is the Eldrich wrench fault. Numerous secondary shear features also accompany it, figure 3. Its strike (close to that of the orebody) is 10°, but the main fault trace varies in dip, 35-45°, and can be steeper locally.

The faulting, besides forming large scale weakness planes, figure 4, is associated with alteration and schistocity. The alteration products in diorite are carbonate/dolomite stringers following the schistocity, and moderate chloritization. In tonalite, the surrounding rock is silicified and moderately hematized. Gouge, up to 45 cm size, of high plasticity, is located on the fault trace. The schistocity, which surrounds the gouge, gradually disappears away from the fault, but can extend 5 m.

The contact between tonalite and diorite is smooth; when the contact is near the roof of the excavation, the back easily detaches itself. The resulting surface is flat and quite extensive, figure 5.

Joint properties are outlined below for the west lenzes sector and zone 5.

The joints intersecting the openings in the vicinity of the west sector are well defined. Four families are evident: NW-SE, subvertical; NNE-SSW, subvertical; NE-SW, 50° NW; subhorizontal, figure 6. All are extensive, usually >2 m, and planar. There is little roughness save the odd small step-like feature, figure 5.

The joint distribution is not regular. The spacing is variable and usually > 30 cm. The occurence of each family is also sporadic, but 2 or more families rarely intersect. When they do, blocks 0.5 - 1 m^3 are formed.

There is also a ubiquitous, poorly developed schistocity parallel to the orientation of the fault. Discontinuities < 60 cm long, spaced 30-50 cm, are evident. On occasion, the rock mass is only intersected by these features.

Joint family predominance in this sector, by rank is: NNE-SSW; NW-SE; NE-SW; subhorizontal.

The structural features of zone 5 will remain largely undefined, until more openings are created. So far, one drift has been excavated in the ore zone.

The drift is cathedral shaped because the fault zone, situated at the hanging wall contact, was removed. The other drift wall showing an orebody section, displays undular joints belonging to the poorly developed schistocity group. These are extensive $\gg$ 2 m, with an average spacing of about 60 cm. The joints are also staggered, figure 7. Few other joint types occur there.

The present method of support in the stopes and drifts are 2.1 m mechanical bolts with the aid of post pillars in the wider stopes. Support is applied immediately after each round of excavation. The widest opening so supported is 30 m wide x 74 m on dip, 4 m high. The stoping and drifting activity performed before 1962 remains unbolted. The largest unbolted openings are stable, reaching 25 m x 30 m, 25 m high, with arched back. There is no evidence of blocks falls, nor is there report of any.

Extraction generally appears to have been done on a once-through basis; secondary extraction does not seem to have been the normal mining practice.

4. ANALYSIS

This report is based on a short underground visit. The following preliminary geomechanical assessment of the projected mining activity of zone 5 is based on limited information.

The drifting in this ore zone has been limited, one cross-cut has been excavated, 2.5 x 2.5 m, and dominated by the occurence of a fault at the hanging wall. No large openings exist. It is obvious that for this sector, intensive field and lab work is necessary to produce a more accurate assessment of the effect of the proposed mining strategy.

In the west lenzes numerous open stopes and various levels already exist. It is thus easier to obtain rock mass characteristics pertinent to future mining activity. Some field and lab work will be required to obtain a clear impression of conditions within future surface crown pillars and the loads that will be imposed on them.

4.1 Mining Strategy

The room and pillar method will be applied to zone 5. It has been identified by the operators as the most economical mining method for this orebody.

Several variations have been considered, although all would use long-hole production drilling, figure 8-10:

- down-dip rib pillars; fan shaped or parallel drilling pattern,
- on-strike pillars; fan shaped drilling.

In the down-dip pillar version, 24 m wide openings would be created, separated by 6 m pillars. The broken ore would be scraped to a mill hole. The company is interested in using another method, such as water jet, rather than scraping the ore.

For the on-strike pillar version, 12 m wide openings would be created, separated by 3 m pillars. The ore would be hauled to an ore-pass system by LHD's.

In both variation stopes are 6 m high; the main drifts are placed at every 46 m or 74 m down-dip.

The mining activity will advance progressively down-dip, but each stope will be excavated from the bottom up.

The method placing rib pillars down-dip and drilling from sublevels is the method preferred at this stage, the mine having virtually abandonned the on-strike version. Pre-split blasting techniques to minimize roof and pillar damage will be applied.

Rock bolting is expected to provide the necessary permanent roof support. As mentioned, backfill will not be used.

With rib pillars placed down-dip, the setting and support structures are similar to Elliot Lake mining activity.

Locating support pillars to accomodate structure is an option considered by the operators. Such planning will have to wait until intensive field work provides sufficient structural indications. The mine plans to create surface crown pillars over some of the west lenzes. So far, the method to expand existing stopes, until surface crown pillars are created, has not been established. But several important stability elements, such as considerable overburden and bodies of water, must be taken into consideration along with rock mass characteristics.

4.2 Rock Mass Quality and Support Requirements

There are several important structural elements with potential influence on roof stability. These are the four joint families, the pseudo schistocity and the fault. But all these elements rarely occur together in the same vicinity; indeed, the arrangement will usually be limited to one joint family accompanying the pseudo schistocity or the fault.

The pseudo schistocity will always require support and attention especially under the large stope spans. Effective bolting, at least on a local scale, will be necessary. Narrow orebodies with planes of weaknesses, stress concentrations and/or large spans can facilitate roof tensile cracking, roof parting or floor heave (2). Because the presence of these parting planes in the roof cannot always be predicted, post blast scaling and roof bolting should be done before production drilling is resumed.

A bolting system which includes sufficient pre-tensioning and anchoring depth, will keep roof slabs, formed by the pseudo-schistocity, tied together. The resulting laminated beam will have greater strength than individual parting layers. In the case of potential block formation, the tensioned bolts will maintain a tight interlocked mass. However, while in most average spans such interlocking blocks can form efficient arching, it is unlikely that efficient arching can be obtained in stopes with large spans.

Ground control will be helped by using careful blasting. A more stable back and minimum pillar damage can be achieved by using pre-split blasting techniques.

A stereographic plot of structural features can be applied in regards to general stability. Gravitational block failures require that a block be separated from the surrounding rock mass by at least three intersecting structural discontinuities. A gravity fall can occur if the stereographic planes surround the centre of the net. Sliding failures will occur if one of the three planes is steeper than the angle of friction.

The stereographic plot in figure 6 displays the structural features likely to be encountered. Although the correct orientation of each family can only be obtained with more field work, these observations can provide a good indication of instability modes.

Roof gravity falls are possible from the intersection of the following families (keeping in mind the variation in dip):

- NW-SE; NNE-SSW; NE-SW 50° NW, (figure 6)
- Pseudo Schistocity (or fault); NW-SE; NNE-SSW, (with or without NE-SW 50° NW),
- Pseudo Schistocity (or fault); NE-SW 50° NW; NW-SE.

The presence of the subhorizontal family, when added to the intersections of three or four joints will be detrimental. Such failures will be rare, as it was observed that the various families rarely intersect.

Side wall failures are also possible. When the extensive, steep dipping joints are parallel to excavation direction, sliding and/or toppling failures can occur. Toppling prisms can also occur. Examples of these were observed during the visit, figures 11-12.

The NW-SE subvertical family is extensive, >2 m. If it is included in down-dip pillars, they will cause pillars to split and weaken. A similar effect on pillars placed on-strike will originate from the presence of NNE-SSW subvertical joints.

The most important stability element at the mine is the fault and its surrounding weakness zone. At the hanging wall contact, it will be very difficult to support because of the gouge and surrounding weak zone. It will be more efficient to bring down these weaknesses until a solid back is formed, such as the case of the existing cross cut in zone 5.

Because it dips 2° more than the orebody this weakness zone will be intersecting all the rib pillars over a down-dip distance of about 170 m. The presence of this zone will weaken the pillars and may introduce shearing or other movements. Furthermore, there is a potential for heave when the fault zone is situated in the immediate floor.

Estimates of rock mass quality and support needs are possible by applying the NGI rock mass classification system (2). Here, the application of this empirical system is based on preliminary information only. When detailed information is available, from intensive field work, this approach can be realistically applied.

The formula to obtain the quality factor "Q" is:

$$Q = \frac{RQD}{Jn} \times \frac{Jr}{Ja} \times \frac{Jw}{SRF}$$

where

RQD is the rock quality designation

Jn is the number of joint sets

Jr is the roughness of the joints

Ja is the joint alteration

Jw is the water condition on the joints

SRF is the stress reduction factor

An estimate of the RQD is made using the field formula:

RQD = 115 - 3.3 Jv

where Jv is the number of joints per cubic meter.

Table 1 outlines the parameters used to obtain an estimate of Q.

The rock mass Q values are moderately high and qualified as good, figure 13. The Q values in the fault zones are low and qualified as very poor.

To relate the quality factor to support requirements, the equivalent dimension factor De is used:

$$De = \frac{\text{span}}{\text{support ratio (ESR)}}$$

In the case of permanent mine openings an ESR of 1.6 is suggested. For a span of 24 m, De = 15.

The prescribed support, figure 13 and table 2, for the rock mass is category 14: systematic, tensioned bolts, 0.5 kg/cm^2 support pressure, 1.5-2 m spacing with chain link mesh. For the fault zone, category 32: systematic, tensioned bolts 3 kg/cm^2 support pressure, 1 m spacing and 40-60 cm mesh reinforced shotcrete. For ceiling bolt lengths the NGI system recommends:

$$2 + 0.15 \frac{\text{(excavation span)}}{\text{ESR}}$$

For the rock mass and fault zone, the suggested length is 2.25m. The anchoring length for the fault should obviously be much more than the thickness of the weak zone. Obviously this becomes impractical for the mine when located immediately above the opening. It is preferable then to let the weakness zone fall during blasting, as the mine has done in the cross-cut of zone 5. The resulting surface is clean and forms a very stable back.

The bolting pattern presently used in the rock mass is a 1.2 m spacing of 2.1 m long bolts.

The bolt length used by the mine appears to be sufficient to anchor the block sizes created by the extensive joints on site. There should be one bolt to support each large block, with sufficient carrying capacity to support that weight.

A future study using numerical modelling will be able to provide an evaluation of which support to use, for both ground types.

4.3 Stress Effects

A mine which relies on pillars for primary support of a large portion of its unfilled openings must be aware of the factors influencing their strength and long term support capability.

In this case the main elements to consider are: natural stresses, induced stresses imposed on pillars as a result of openings created, weakening elements included in the volume of rock (in and surrounding the pillar) and strength of the intact rock.

The main features to consider in analysing stress effects are: direction and magnitude of in-situ principal stresses, orientation of the opening within the stress field and opening geometry.

The proposed stope and pillar layout at the Eldrich mine is simple: consecutive stopes, 24 meters wide interspaced with 6m wide rib pillars. The deposit is 160m wide at its largest extent, necessitating five or six stopes of the stated arrangement. In either case stress orientation and displacement would be accurately modelled by computer methods discussed in section 5. Two simple approaches will be used to examine pillar loads under these conditions. The stress level existing in the shallowest rib pillars (depth 375 m) will be given as a function of gravity loading (lower bound) and stress superposition effects (upper bound).

The tributary area theory (3) identifies the average pillar stress as:

$$\sigma_p = \frac{1}{1 - R} S_o$$

where

S_o = uniaxial field stress

R = extraction ratio

In the case of inclined pillars, S_o must be converted to reflect horizontal and vertical in-situ stress components.

Vertical loading at 375 m, due to gravity is 10.2MPa. The horizontal stress, assuming elastic conditions and a Poisson's ratio of 0.22 is:

$$\sigma_h = \sigma_v \left\{\frac{\nu}{1 - \nu}\right\} = 2.9 \text{ MPa}$$

The effect of the vertical and horizontal stresses are:

$$S_o = \sigma_v \cos^2 \alpha + \sigma_h \sin^2\alpha$$

where α = orebody dip, 38° at Eldrich. The value for S_o becomes 7.4 MPa. R = 0.83, therefore σ_p = 44.4 MPa.

Higher pillar loads would be anticipated using values normally found in the Canadian Shield. The ground stresses are larger horizontally than vertically. Herget (4) has established that the principal stresses are oriented about parallel and at right angles to the earth's surface. The following stress relationships were calculated:

$$\sigma_{vertical} = 0.0260 \text{ to } 0.324 \text{ MPa per metre depth}$$

$$\kappa_{max} = \frac{\sigma \text{ horiz max}}{\sigma \text{ vertical}} = \frac{253.87}{\text{depth (m)}} + 1.45$$

$$\kappa_{min} = \frac{\sigma \text{ horiz min}}{\sigma \text{ vertical}} = \frac{279.72}{\text{depth (m)}} + 0.88$$

Assuming a major principal stress oriented E-W (which is often the case), would indicate a principal stress orientation parallel and perpendicular to rib pillar axis. This setting will form the input to analysing pillar loads based on planned dimensions and the superposition effects of consecutive stopes.

The procedure will be to consider a single opening, using elastic analysis, with the Geldart and Udd approach (5). Stress orientations as a function of the number of pillars and the ratio of opening width to pillar width will then be used to obtain representative pillar loads.

The tangential stress imposed on an opening surface is given by:

$$\frac{\sigma_\eta}{S_y} = \frac{2\nu(1+\kappa) + (1-\kappa)(1-\nu^2)\cos 2\beta + (1-\kappa)(1+\nu)^2\cos 2(\beta-\eta)}{(1+\nu^2) + (1-\nu^2)\cos 2\eta}$$

where

σ_η = tangential stress on the ellipse surface

ν = semi-major axis / semi-minor axis = axis ratio

k = S_x/S_y = pre-mining horizontal / vertical stress = stress ratio

β = angle clockwise from semi-major axis to S_x

η = elliptical co-ordinate measurement clockwise from the semi-major axis

The compressive pillar stress in this single opening case, figure 14 and table 3, is 139.4 MPa. The mid-span is subjected to important tensile stresses, 3.8 MPa and the pillar-roof corner is compressively stressed, 74.6 MPa, which may impose shearing movements. Considering the effects of five consecutive openings, the superposition of stresses from adjoining stopes, because of the narrow pillars, will magnify the load distribution calculated for a single opening.

The 44 to 139 MPa range is the minimum pillar load range likely, the latter figure being more representative of Canadian Shield conditions. Given the role they play in maintaining stability in open conditions (along with mechanical rock bolts), these pillars must be strong enough to fulfill the role expected of them. This could be permanent support or gradual failure after local mining has finished.

Pillars can fail in several fashions; if the pillars are massive enough, rather than by weakening along structure planes, high compressive stresses can lead to splitting, hourglassing or crushing (if the rock is of high strength). The mine, however, cannot afford pillar degradation during mine life under such large spans; it would lead to serious loss of support capacity. Akin to this limitation, the mine cannot afford to carryout actual pillar failure tests to determine pillar strength. Instead it has to depend on lab tests on various specimen sizes and back-analyse stable pillars to provide strength values. The serious drawback with this method is that lab samples provide a higher strength than actually encountered in the field.

This problem was addressed successfully by Hedley (6) and Herget et al (7) who performed lab tests on specimens of various sizes and extrapolated the results to pillar size to obtain typical strength values respectively for Elliot Lake and Sudbury mine pillars. Hedley also combined this to back analysis of stable and unstable mine pillars. This information was then incorporated into a safety factor analysis between strength and induced stress, producing design curves for pillars. Such design could also be applied for the Eldrich mine, once accurate in-situ stress determinations are made.

Of particular concern for the mine is the high values in tensile stress imposed on the roof of the openings. Mechanical anchors there will have to be deep enough with sufficient tension to maintain a locked mass.

The mine plans to reach a depth of at least 650 m. There, the vertical and horizontal stresses with the same pillar orientation would be 18.0 MPa and 23.6 MPa respectively, figure 15. The single opening stresses for pillars, roof mid-span and corner would be much greater than at 375 m, respectively 194 MPa, 3 MPa and 107 MPa, table 3. Stresses of this magnitude would be too great for the pillar. A 50% reduction in strength with an increase in diameter from lab to field is a typical but not universal order of magnitude. The strength of the tonalite or diorite from lab tests will probably indicate that a load of 134 MPa is better suited for these rocks than one of 194 MPa, in the light of the unconfined compressive strength range this rock is likely to possess. The mining method or extraction ratio will have to be reviewed for the lower portion of the mine.

There is another effect from in-situ stresses the mine will have to consider. There will exist the possibility of stress concentration as the mine deepens and reaches the boundary with a coarser grained, intrusive rock, around the 700 m depth.

5. FUTURE STABILITY STUDIES

Because the mine is at a pre-excavation stage, it is important to obtain as much pertinent information as possible to examine a variety of mine layouts and mining sequences and arrive at a stable and economic mining strategy. Furthermore, obtaining as much information as possible will avoid major unexpected problems. Since the mine's extraction ratio is high ~ 83% and since the use of backfill to stabilize openings is uneconomical, the loss of bearing capacity cannot be afforded.

In mining zone 5, there are several caveats to consider: stability of a laminated stope roof, faulted and weakened support pillars, narrow pillars, wide openings, stress concentrations, detrimental joint intersections.

A suitable stability research project must address these considerations. The components for such a study of the Eldrich Mine are:

1. coring and structural data gathering;
2. in-situ and laboratory determination of strength and deformation properties;
3. numerical modelling;
4. site monitoring.

Comprehensive core drilling and structural data gathering programs are needed to evaluate local and general rock quality. It becomes important to carry out mapping of existing openings and drilling at various locations under different orientations to collect pertinent geomechanical data, where none had been collected before. This will serve several purposes under the planned mining strategy: identify support requirements, improve structural geology conditions for the pillars and provide the operating personnel with a "feel" for the rock mass in which they are operating.

The core will be used to test for rock strength and deformation characteristics required in evaluating pillar support capabilities and roof/floor behaviour. Furthermore, the broken condition of the pillars can be evaluated and provide another indication of strength capability. The strength carried by pillars, when beyond supporting gravity load only, is difficult to assess. Just as difficult is the value to adopt from laboratory test on rock core for design of openings. There exists several empirical approaches to transpose lab results. A choice of which to adopt is easier when numerical modelling and further evaluation of structure and rock mass quality are done. As well, the structural record of the core will provide joint characteristics to complement structural geology field work and thus arrive at values to be used in the NGI classification. The NGI support recommendations seem apt to be used here.

To represent variations in mass properties, it is important that core be obtained from each different rock mass sector: tonalite, diorite, fault material, surrounding schistocity. If core is obtained from hangingwall and footwall regions, such a general impression of the surrounding pseudo-schistocity/laminations will provide information to the modeller in the form of general weakness trends and local considerations.

Special testing, of fault gouge, schist or other weak pillar material is especially important, not only in the sense of quantifying reduction in pillar strength, but properly representing possible displacements with the help of numerical modelling.

The structural elements so far are only marginally known and therefore not well defined. Mapping traces of major and common features will help the mine staff and modellers in deciding on pillar and stope dimensions and location. Roof classifications, with the help of a well developed system of characteristics, will identify the relative effects of various structures and their area of influence in the stopes. Depth in the hanging wall, extent and degree of development for example can all be used in a relative rating, i.e. poor to excellent.

In a future study of the Eldrich mine, finite element modelling will be a very important element of design, in determining stress concentrations and the stability of openings in multiple openings settings. It can well represent each of the distinct rock mass zones encountered there. It is a flexible method in that various mine plans and geometries can be modelled and evaluated. By studying various models, the best mining sequence to avoid high stress concentrations and ground control problems can be established.

One of these sequences, worth considering, is one where mining progresses from the deepest reaches to the shallowest extent of the orebody and leaves behind thin pillar that would yield after the mining sequence has reached shallower ground away from yielding pillars. This might successfully address two ground control problems, that of maximizing extraction while maintaining safe mining conditions and avoiding serious stress conditions. But it should not lead to uncontrolled pillar collapse and overall instability. Barrier pillars may be required to act as an abutment and regional support, isolating the existing works from the new development.

Within the Eldrich setting, whether the pillars are planned to stand or yield, the existence of regularly placed pillars introduces the concept of load sharing and the long term behaviour of those likely to carry redistributed loads after weakening of some.

For various mining strategies, the mine can perform a cost analysis on the economics of varying pillar size vs. artificial support costs. In this respect, numerical modelling can be of great use.

In situ stress measurements would reveal the orientation and magnitude of the major principal stresses. This would provide realistic values to use in modelling and other analyses. Though average values can be used, this could lead to over/underestimation of loads by assuming no anomalous stress conditions exist at the mine site.

The mine is highly advised to monitor pillar stresses and pillar movements, especially in the area where the fault zone will transect the rib pillars and at deeper reaches, where stresses may prove to high for the pillar strength. As well, the presence of dikes/sills, which may be stress concentrators, should be identified. Monitoring of these also becomes important.

Laboratory tests will go a long way to establish basic strength parameters. This includes shear and compressive strength of fault material, uniaxial and tensile strengths and extrapolation of rock mass strength based on the Hoek and Brown criteria (8).

Roof monitoring in such geological material is important. Roof stability is necessary for two reasons: to prevent dilution by waste rock and to maintain the stope in operating condition. The relative displacements of selected points in a borehole oriented vertically are required. Extensometers with multiple anchors would be adequate. The instrument can be installed in holes that provide the rock core. Extensometer location should be in the mid-span area of the stope hangingwall. Floor to ceiling convergence measurements will not be adequate due to the activity of the ore scrapers and blast flyrock. Just as important is a regular visual inspection of roof and pillar conditions and noting of progression and location of failure conditions as well as general behaviour as mining progresses.

In particular, movements along or across the fault are extremely important to assess the general stability of the upper stopes in zone 5.

Information from monitoring observations can then be reintroduced into modelling/strength property analyses for readjustments.

Secondary extraction of rib pillars, in whole or in part, is worth studying and planning for, but it is likely that back instability conditions rather than pillar overstress will determine the feasibility of this operation. Artificial pillars such as timber packs or concreted mill liners could be used to ensure temporary stable back conditions during the period that workers are exposed.

In summary there are three mining approaches to consider if fill is not used. Mining at the projected extraction ratio, leaving pillars as mined after primary extraction. This is the safest method of the three but the least economical. The second would follow the planned extraction ratio but recover some rib pillars entirely or remove part of each pillar thereby leaving behind "post" pillars. This would entail a constant pillar and back stabilization program. The third option is to retreat up-dip, mining at a higher primary extraction ratio and creating pillars intended to yield after mining activity has sufficiently progressed up-dip for a miner safety and avoidance of high pillar stresses in active stoping areas.

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Table 1: ESTIMATION OF Q VALUES FOR THE TWO ELDRICH MINE SECTORS

	WEST LENZES	WEST END FAULT ZONE	ZONE 5	ZONE 5 FAULT ZONE
Jv	6	9	4	4
RQD	95	40*	100	40*
Jn	9	12	3	3
Jr	1.5	1.0	1.0	1.0
Ja	1.0	13.0	1.0	13
Jw	1.0	1.0	1.0	1.0
SRF	1.0	2.5	1.0	2.5
	Q = 15.8	Q = 0.10	Q=33.3	Q = 0.41

* based on field observations

Table 2. (a) support measures for rock mass quality categories 1-16 (see figure 13), explanatory notes in table 2 (c) (2).

Support category	Q	Conditional factors RQD/J_n	J_r/J_n	$SPAN/ESR$ (m)	P kg/cm² (approx.)	$SPAN/ESR$ (m)	Type of support	Note see p. 229
1*	1000—400	—	—	—	< 0.01	20—40	sb (utg)	—
2*	1000—400	—	—	—	< 0.01	30—60	sb (utg)	—
3*	1000—400	—	—	—	< 0.01	46—80	sb (utg)	—
4*	1000—400	—	—	—	< 0.01	65—100	sb (utg)	—
5*	400—100	—	—	—	0.05	12—30	sb (utg)	—
6*	400—100	—	—	—	0.05	19—45	sb (utg)	—
7*	400—100	—	—	—	0.05	30—65	sb (utg)	—
8*	400—100	—	—	—	0.05	48—88	sb (utg)	—
9	100—40	≧ 20	—	—	0.25	8.5—19	sb (utg)	—
		< 20	—	—			B (utg) 2.5—3 m	—
10	100—40	≧ 30	—	—	0.25	14—30	B (utg 2—3 m	—
		< 30	—	—			B (utg) 1.5—2 m + clm	—
11*	100—40	≧ 30	—	—	0.25	23—48	B (tg) 2—3 m	—
		< 30	—	—			B (tg) 1.5—2 m + clm	—
12*	100—40	≧ 30	—	—	0.25	40—72	B (tg) 2—3 m	—
		< 30	—	—			B (tg) 1.5—2 m + clm	—
13	40—10	≧ 10	≧ 1.5	—	0.5	5—14	sb (utg)	I
		≧ 10	< 1.5	—			B (utg) 1.5—2 m	I
		< 10	≧ 1.5	—			B (utg) 1.5—2 m	I
		< 10	< 1.5	—			B (utg) 1.5—2 m + S 2—3 cm	I
14	40—10	≧ 10	—	≧ 15	0.5	9—23	B (tg) 1.5—2 m + clm	I, II
		< 10	—	≧ 15			B (tg) 1.5—2 m + S (mr) 5—10cm	I, II
		—	—	< 15			B (utg) 1.5—2 m + clm	I, III
15	40—10	> 10	—	—	0.5	15—40	B (tg) 1.5—2 m + clm	I, II, IV
		≦ 10	—	—			B (tg) 1.5—2 m + S (mr) 5—10cm	I, II, IV
16* See note XII	40—10	> 15	—	—	0.5	30—65	B (tg) 1.5—2 m + clm	I, V, VI
		≦ 15	—	—			B (tg) 1.5—2 m + S (mr) 10—15cm	I, V, VI

* Authors' estimates of support. Insufficient case records available for reliable estimation of support requirements.

The type of support to be used in categories 1 to 8 will depend on the blasting technique. Smooth wall blasting and thorough barring-down may remove the need for support. Rough-wall blasting may result in the need for single applications of shotcrete, especially where the excavation height is > 25 m. Future case records should differentiate categories 1 to 8.

Key to Support Tables:

sb = spot bolting
B = systematic bolting
(utg) = untensioned, grouted

Table 2. (b) support measures for rock mass quality categories 29-32 (see figure 13), explanatory notes in table 2 (c) (2).

29*	0.4—0.1	> 5	> 0.25	—	3.0	1.0—3.1	B (utg) 1 m + S 2—3 cm	—
		≦ 5	> 0.25	—			B (utg) 1 m + S (mr) 5 cm	—
		—	≦ 0.25	—			B (tg) 1 m + S (mr) 5 cm	—
30	0.4—0.1	≧ 5	—	—	3.0	2.2—6	B (tg) 1 m + S 2.5—5 cm	IX
		< 5	—	—			S (mr) 5—7.5 cm	IX
		—	—	—			B (tg) 1 m + S (mr) 5—7.5 cm	VIII, X, XI
31	0.4—0.1	> 4	—	—	3.0	4—14.5	B (tg) 1 m + S (mr) 5—12.5 cm	IX
		≦ 4, ≧ 1.5	—	—			S (mr) 7.5—25 cm	IX
		< 1.5	—	—			CCA 20—40 cm + B (tg) 1 m	IX, XI
		—	—	—			CCA (sr) 30—50 cm + B (tg) 1 m	VIII, X, XI
32 See note XII	0.4—0.1	—	—	≧ 20 m	3.0	11—34	B (tg) 1 m + S (mr) 40—60 cm	II, IV, IX, XI
		—	—	< 20 m			B (tg) 1 m + S (mr) 20—40 cm	III, IV, IX, XI
		—	—	—			CCA (sr) 40—120 cm + B (tg) 1 m	IV, VIII, X, XI

* Authors' estimates of support. Insufficient case records available for confident prediction of support requirements.

Table 2. (c) explanatory notes for table 2 (a) and (b) (2).

I. For cases of heavy rock bursting or "popping", tensioned bolts with enlarged bearing plates often used, with spacing of about 1 m (occasionally down to 0.8 m). Final support when "popping" activity ceases.

II. Several bolt lengths often used in same excavation, i. e. 3, 5 and 7 m.

III. Several bolt lengths often used in same excavation, i. e. 2, 3 and 4 m.

IV. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 2—4 m.

V. Several bolt lengths often used in some excavations, i.e. 6, 8 and 10 m.

VI. Tensioned cable anchors often used to supplement bolt support pressures. Typical spacing 4—6 m.

VII. Several of the older generation power stations in this category employ systematic or spot bolting with areas of chain link mesh, and a free span concrete arch roof (25—40 cm) as permanent support.

VIII. Cases involving swelling, for instance montmorillonite clay (with access of water). Room for expansion behind the support is used in cases of heavy swelling. Drainage measures are used where possible.

IX. Cases not involving swelling clay or squeezing rock.

X. Cases involving squeezing rock. Heavy rigid support its generally used as permanent support.

XI. According to the authors' experience, in cases of swelling or squeezing, the temporary support required before concrete (or shotcrete) arches are formed may consist of bolting (tensioned shell-expansion type) if the value of RQD/J_n is sufficiently high (i. e. >1.5), possibly combined with shotcrete. If the rock mass is very heavily jointed or crushed (i. e. $RQD/J_n < 1.5$, for example a "sugar cube" shear zone in quartzite), then the temporary support may consist of up to several applications of shotcrete. Systematic bolting (tensioned) may be added after casting the concrete (or shotcrete) arch to reduce the uneven loading on the concrete, but it may not be effective when $RQD/J_n < 1.5$, or when a lot of clay is present, unless the bolts are grouted before tensioning. A sufficient length of anchored bolt might also be obtained using quick setting resin anchors in these extremely poor quality rockmasses. Serious occurrences of swelling and/or squeezing rock may require that the concrete arches are taken right up to the face, possibly using a shield as temporary shuttering. Temporary support of the working face may also be required in these cases.

XII. For reasons of safety the multiple drift method will often be needed during excavation and supporting of roof arch. Categories 16, 20, 24, 28, 32, 35 ($SPAN/ESR > 15$ m only).

XIII. Multiple drift method usually needed during excavation and support of arch, walls and floor in cases of heavy squeezing. Category 38 ($SPAN/ESR > 10$ m only).

TABLE 3

TANGENTIAL STRESSES AROUND A SINGLE OPENING,

ELDRICH MINE†

		$\underline{\eta = 0°}$ (pillar mid height)	$\underline{\eta = 13°}$ (pillar-roof corner)	$\underline{\eta = 90°}$ (roof mid-span)
Width	24m			
Height	6m			
Depth	375m	139.4 MPa	74.6 MPa	-3.8 MPa*
ν	4			
κ	1.63			
β	0°			
Width	24m			
Height	6m			
Depth	650m	194.4 MPa	107.0 MPa	3.4 MPa
ν	4			
κ	1.31			
β	0°			

† The stress distributions are portrayed in Figures 15, 16.

* A negative value indicates tensile stress.

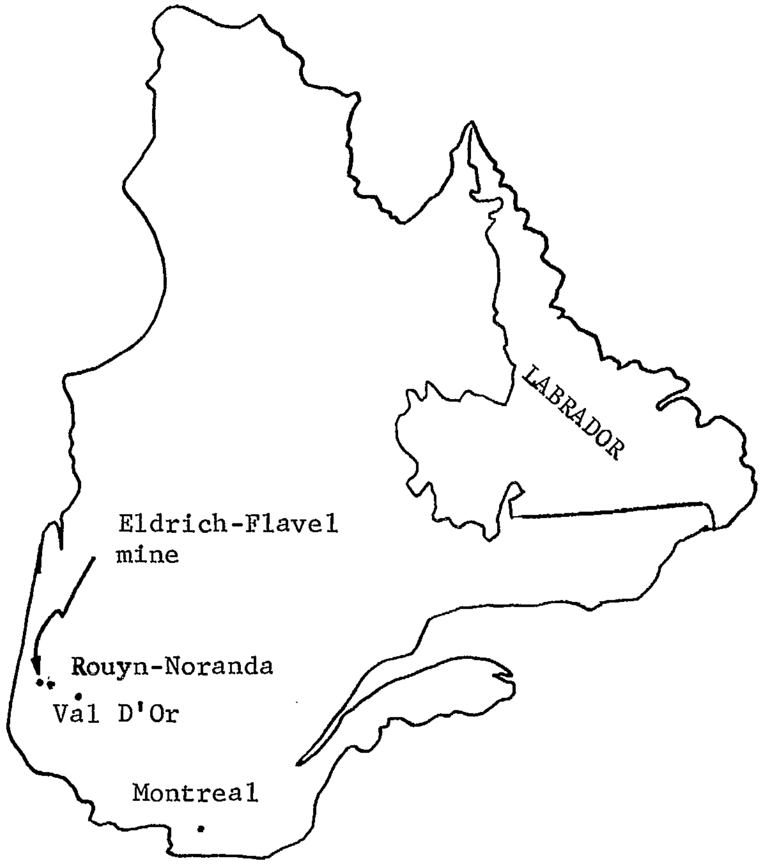


Figure 1 Location of the Eldrich-Flavel mine, province of Quebec.

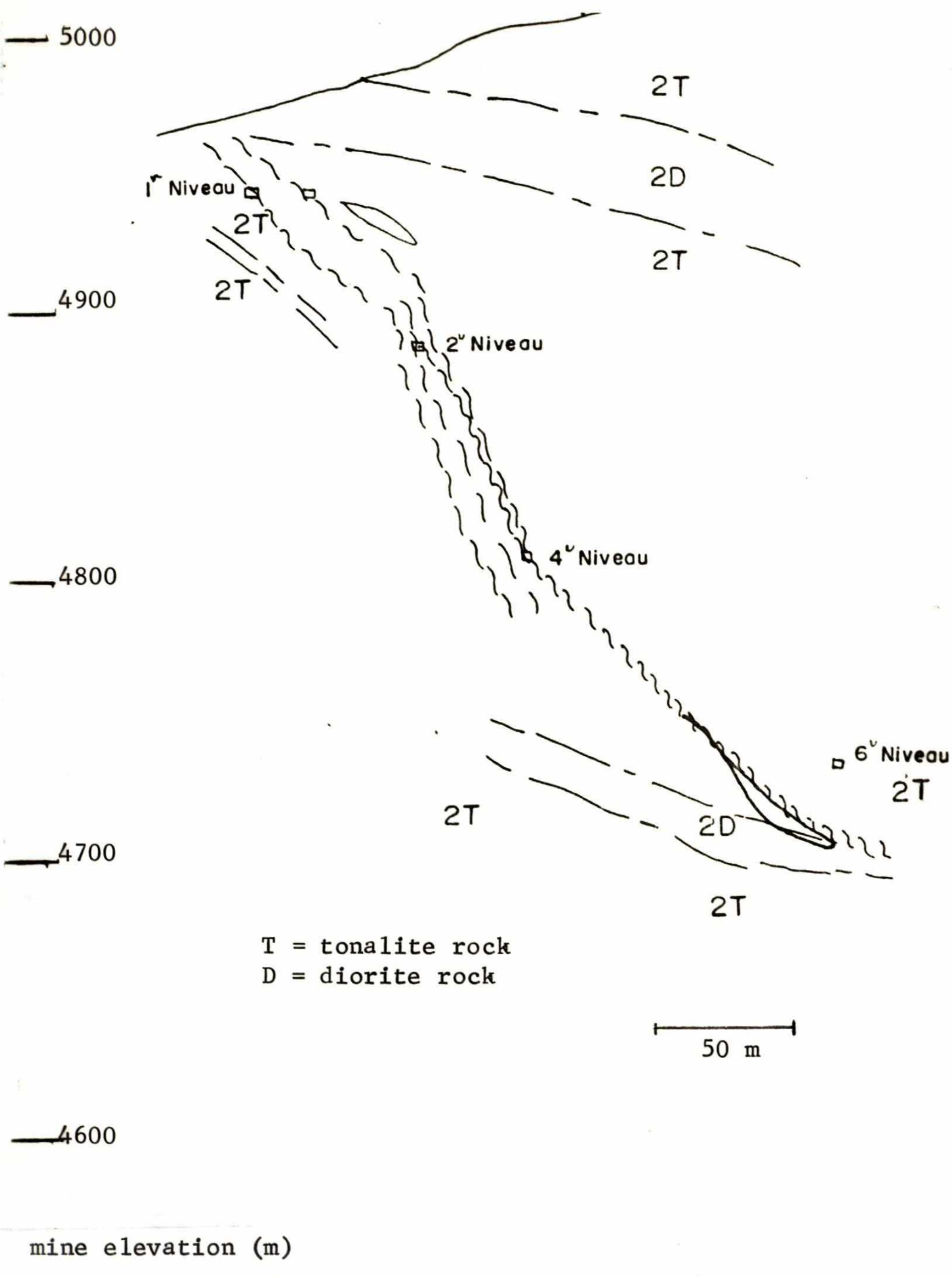


Figure 3. Typical cross-section of mineralization at the Eldrich-Flavel mine. Note the presence of a main fault trace and secondary shear features (mine plan).

Figure 4. Fault zone contact, level 6 zone 5. The fault gouge and surrounding schist have been removed from the hanging-wall leaving nehind this smooth, extensive back. The person on the right hand side has his hand on the fault gouge.

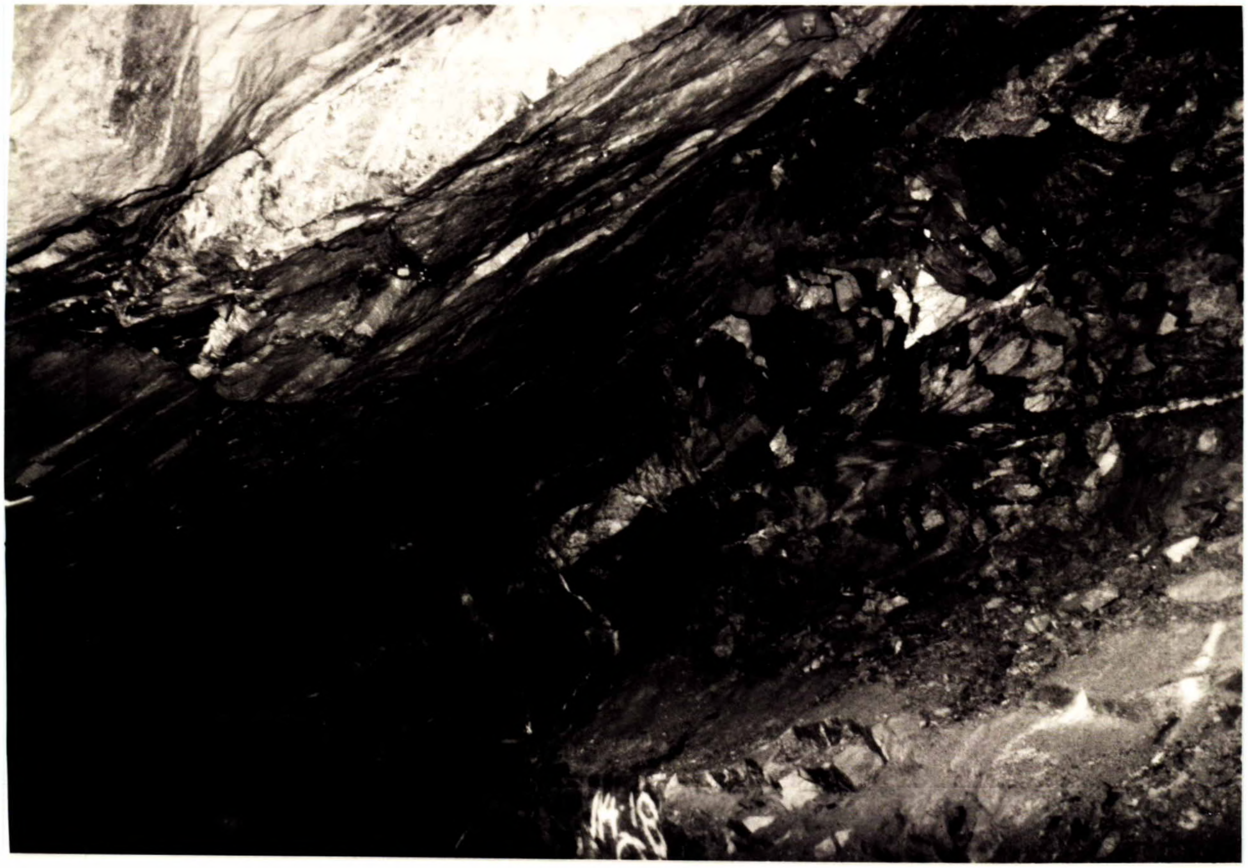


Figure 5. On-strike view of an open stope. The hangingwall surface represents easily detachable tonalite-diorite contact. Notice the "pseudo-schistocity" joints in all portions of the rock mass. These are undulating the others flat. All are smoothsave for the odd step-like features. The bolting pattern is 1.2 x 1.2 m.

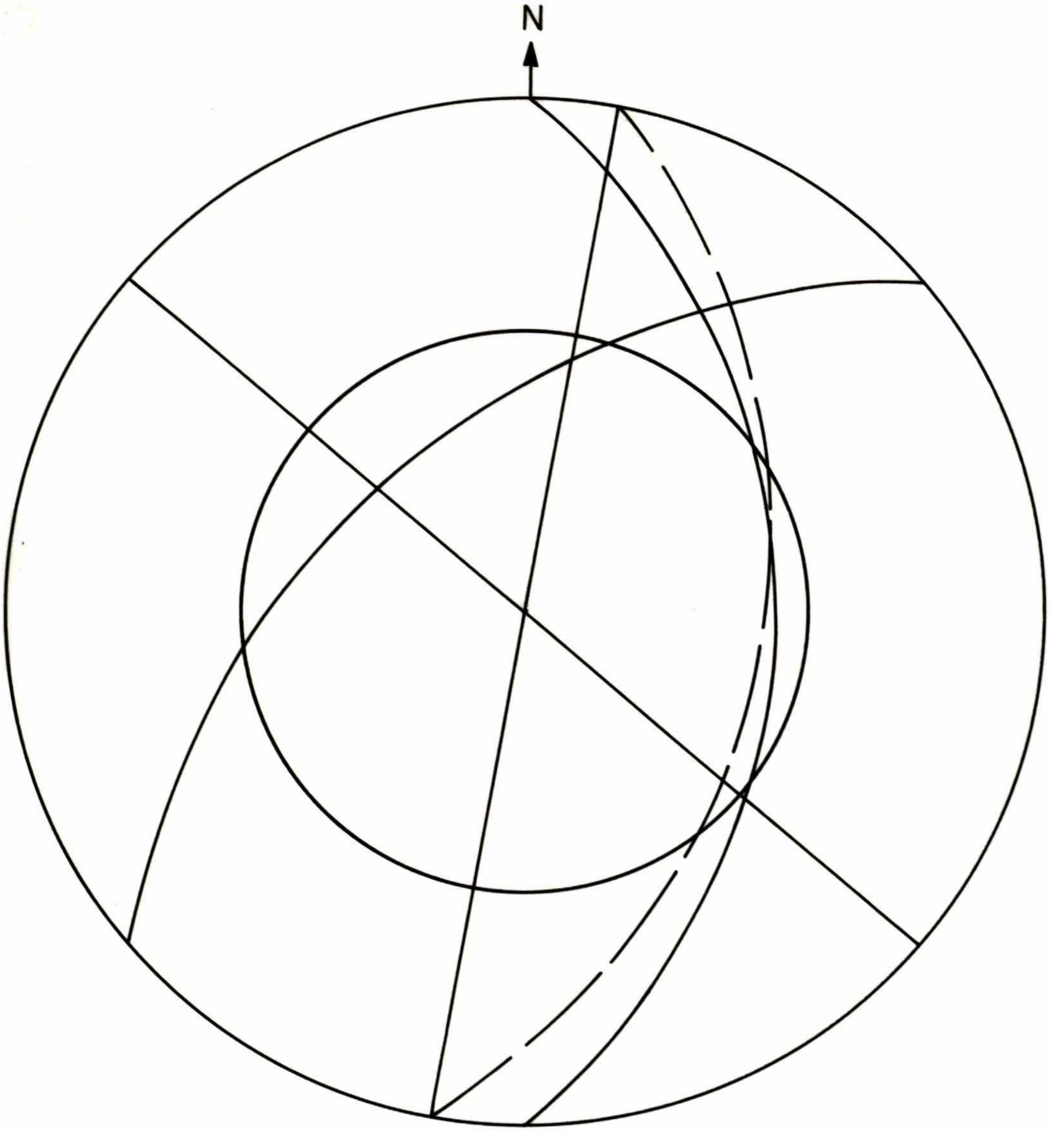


Figure 6. Stereographic plot of structural features likely to be encountered. The great circles represent approximations of actual strike and dip values. The interior circle represents a conservative approximation of the angle of internal friction (34°) for the rock types (tonalite, diorite) around the mine openings. The dashed great circle indicates the fault trace. Lower hemisphere projection.

Figure 7. Orebody below the fault zone, opposite the side pictured in figure 4. Note the absence of joints except for the undulating "pseudo-schistocity".

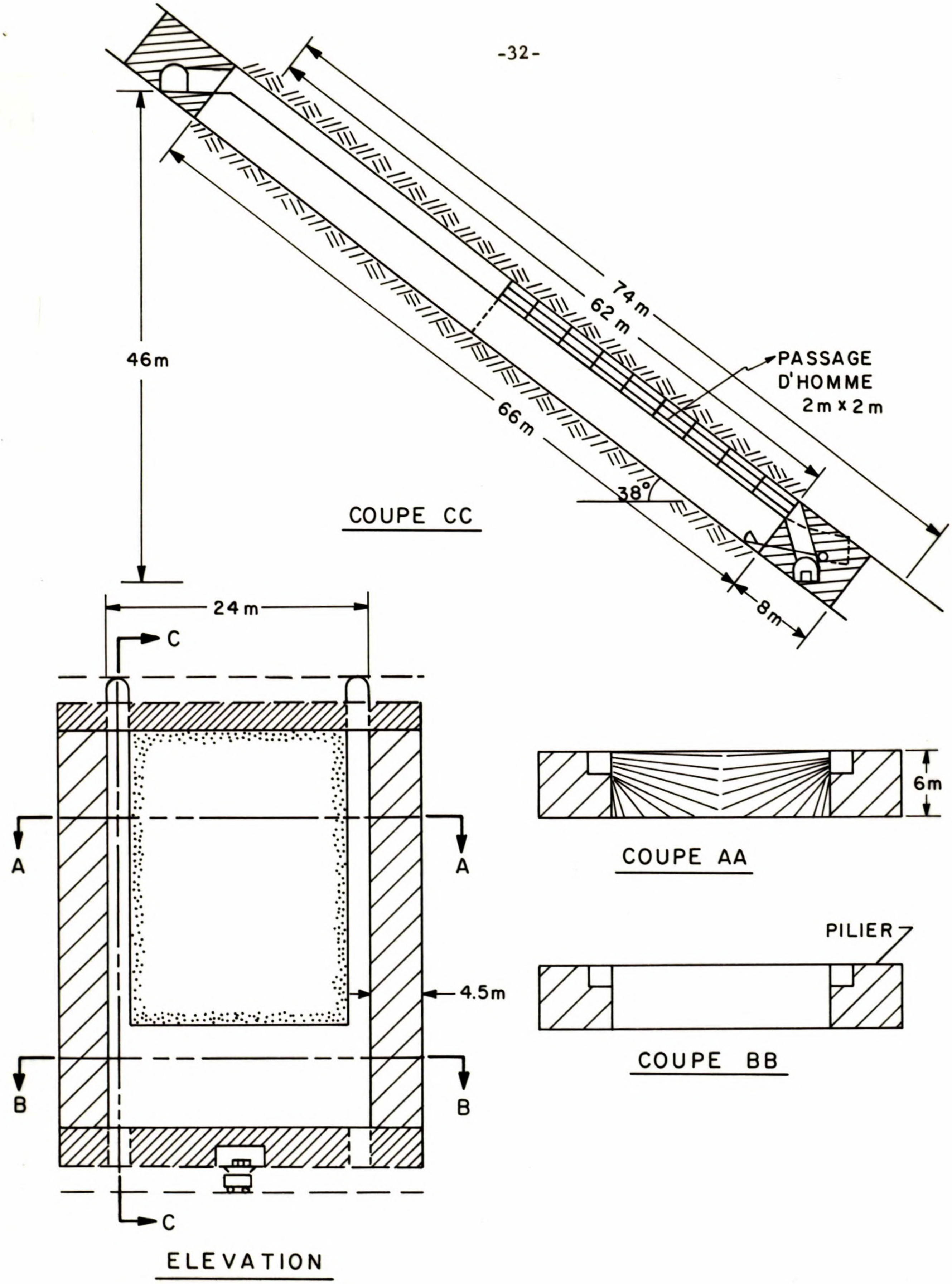


Figure 8. Possible extraction method, zone 5, Eldrich-Flavel mine. Full-height progressive up-dip slicing using fanned long-holes drilled from side raises (mine plan).

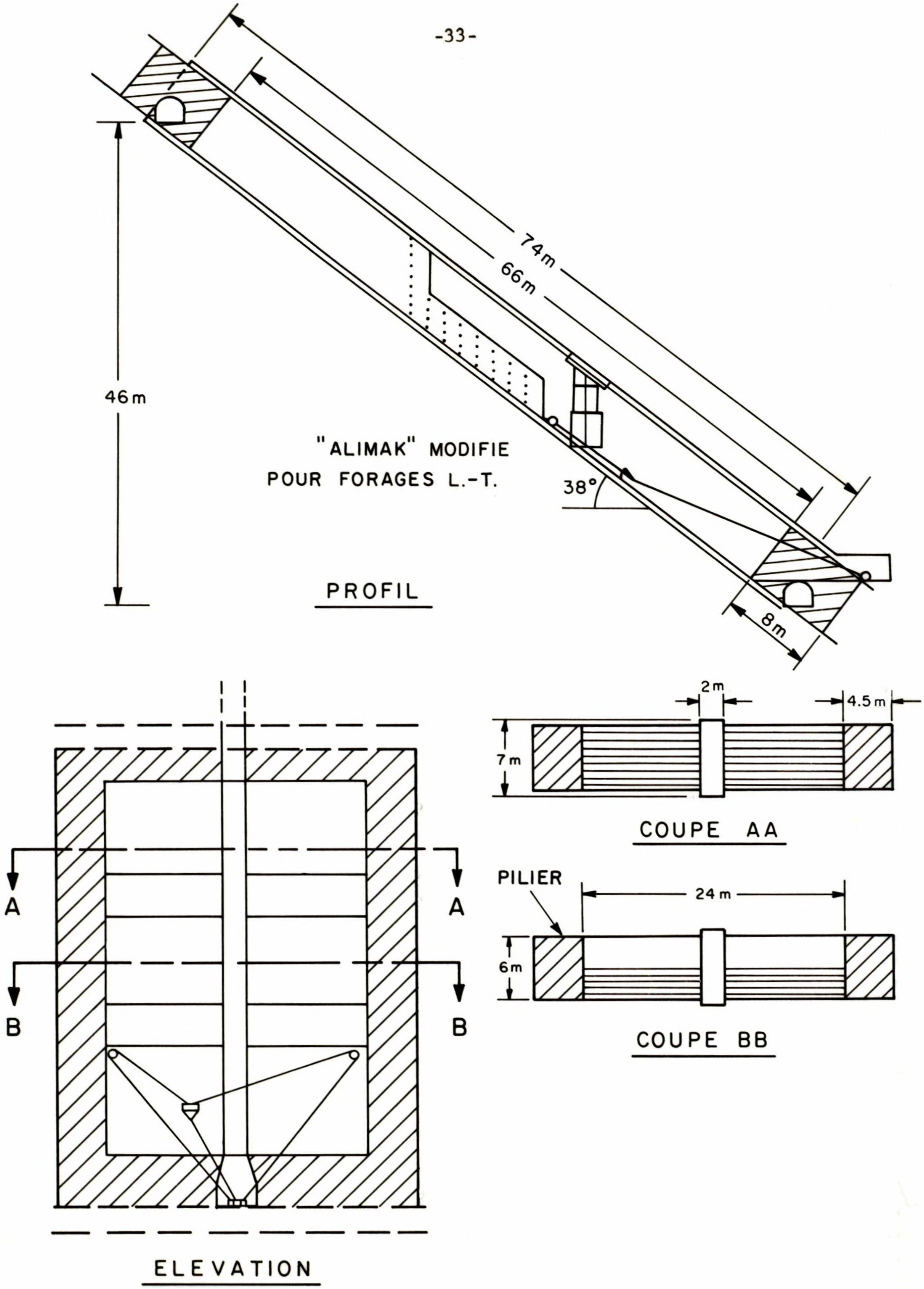
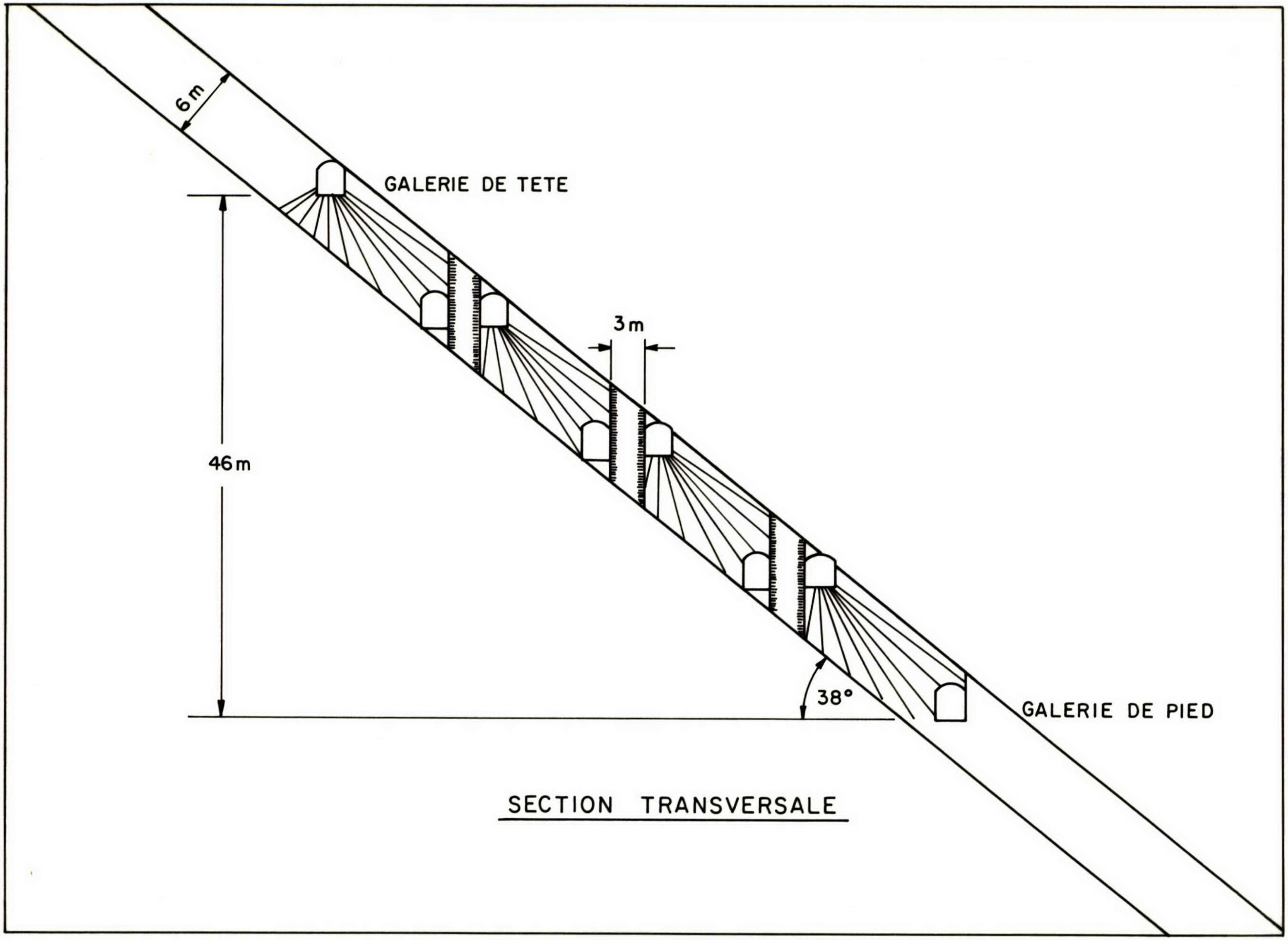
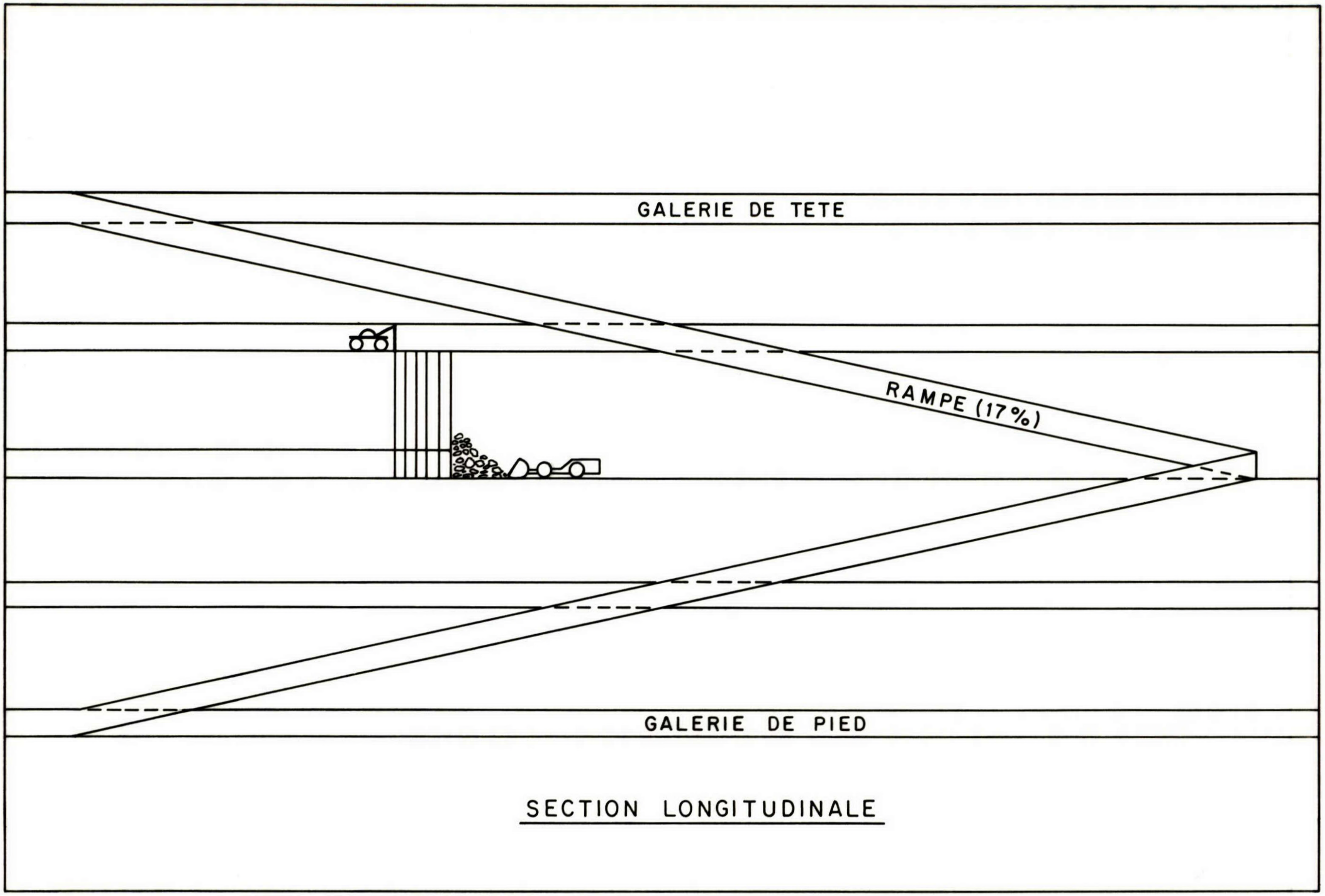


Figure 9. Possible extraction method zone 5, Eldrich-Flavel mine. Half-height progression up-dip slicing using parellel long holes drilled on either side of a central raise (mine plan).



(a)

Figure 10. Possible method of extraction zone 5, Eldrich-Flavel mine. Longitudinal advance using fanned longholes drilled from sublevels. (a) transversal sectio (b) longitudinal section (mine plans).


GALERIE DE TETE
RAMPE (17%)
GALERIE DE PIED
SECTION LONGITUDINALE

(b)

Figure 11 Potential side wall toppling failures. The opposite wall would be subjected to sliding failures.

Figure 12 Potential toppling prism failures.

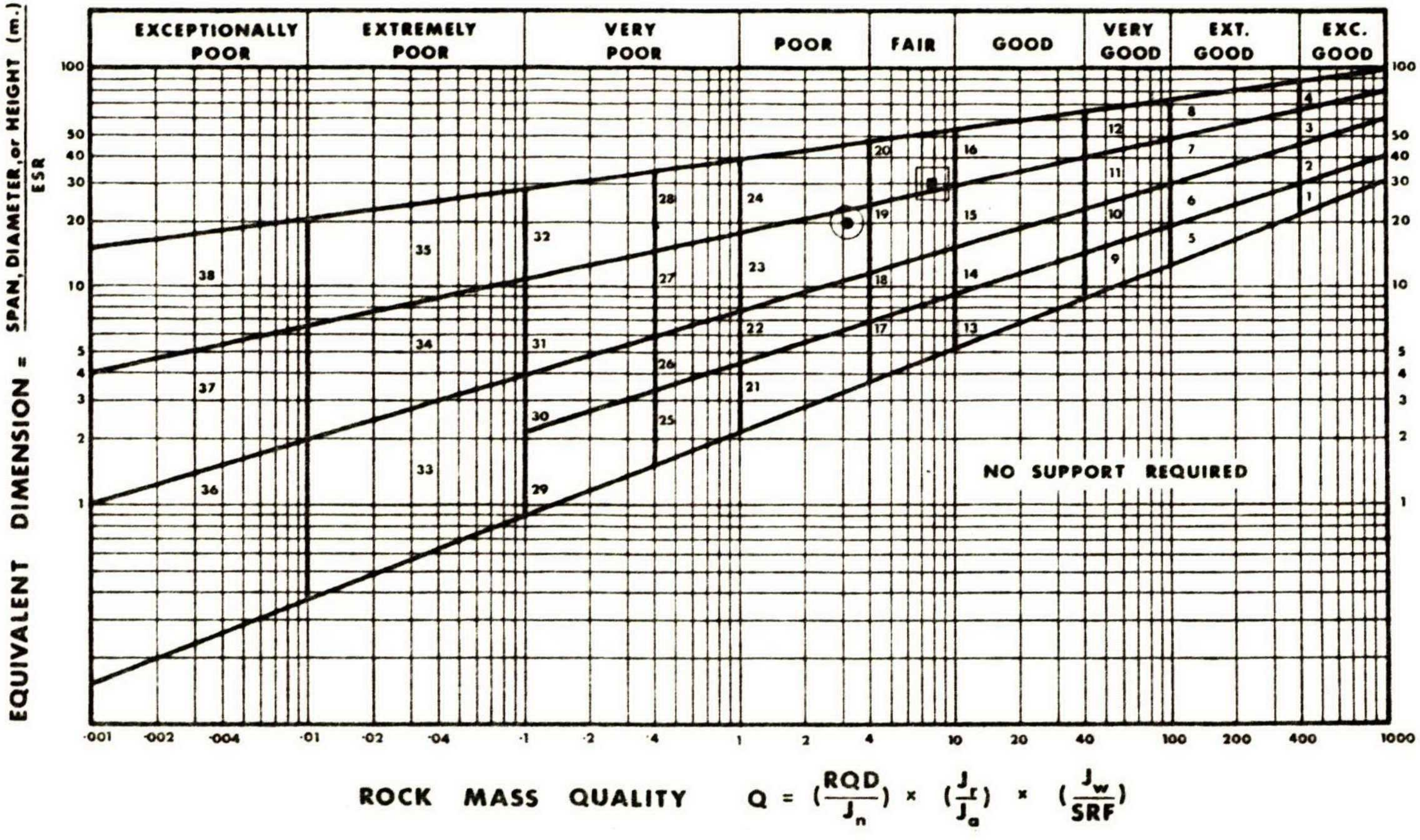


Figure 13. Range of Q values and related qualifications. Support categories with Q and equivalent dimension values (see text) (2).

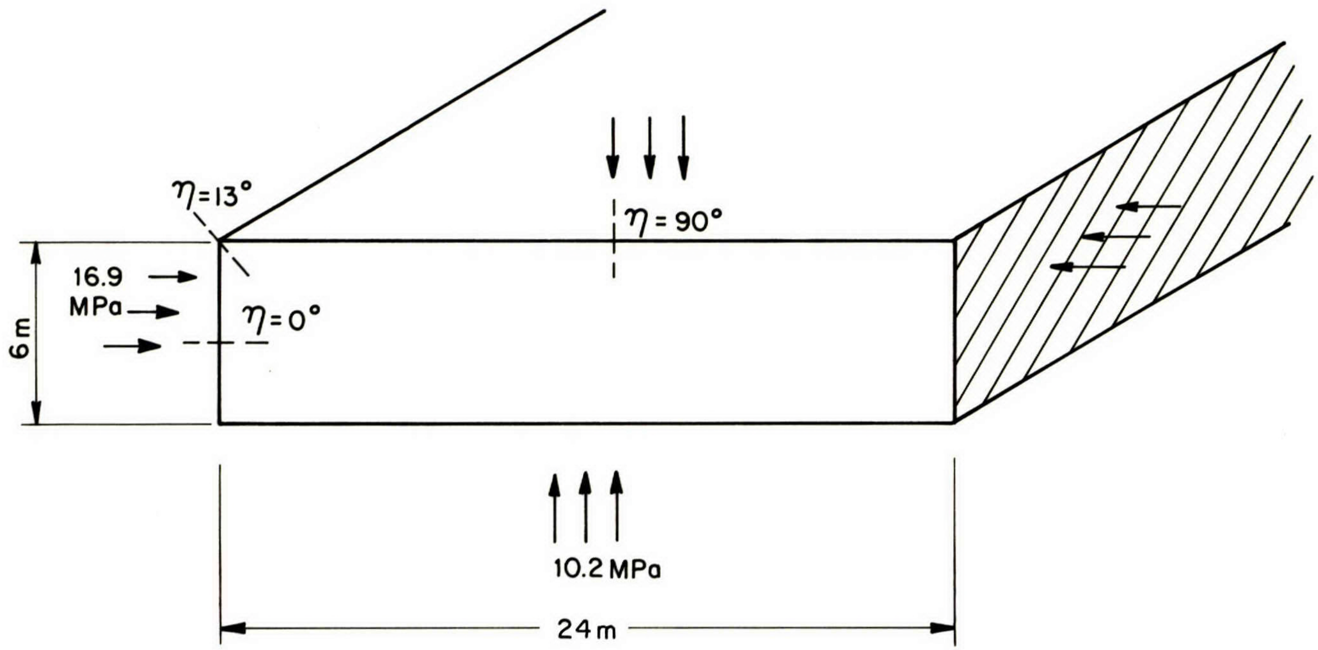


Figure 14. Ground stresses for a single opening at a depth of 375 m, Eldrich Mine. The values of tangential stresses for the indicated locations around the opening ($\eta = 0^{\circ}$, 13°, 90°) are given in Table 3.

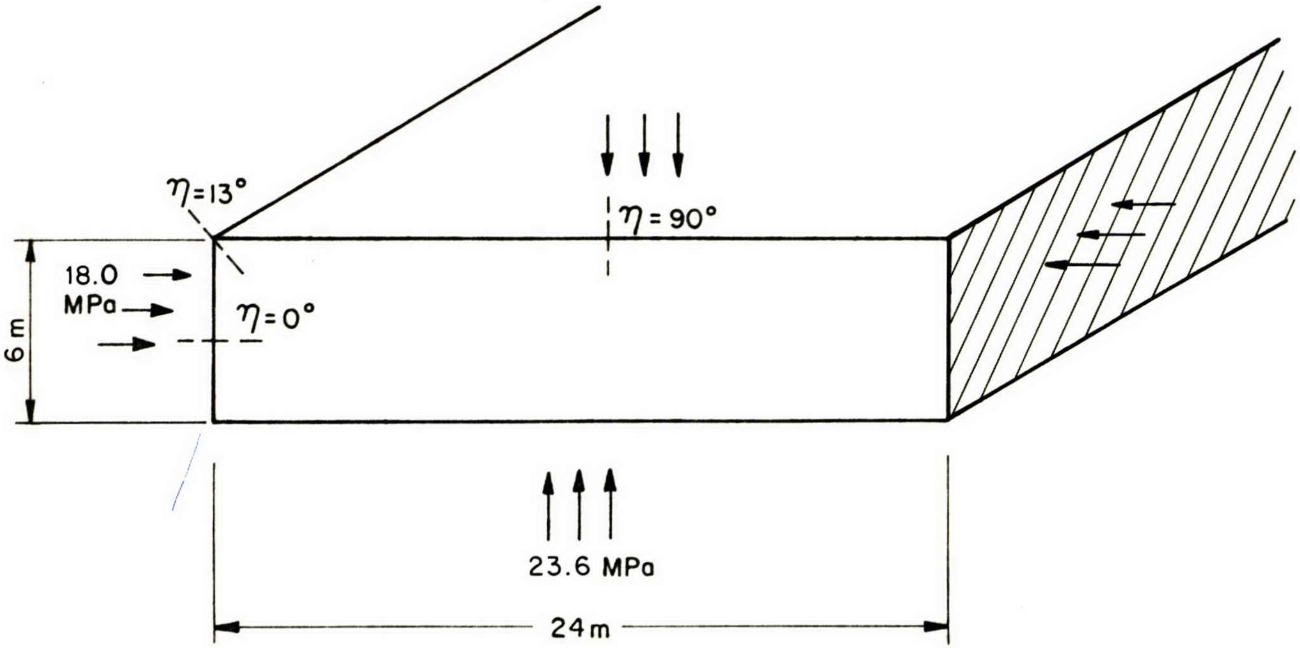


Figure 15. Ground stresses for a single opening at a depth of 650 m, Eldrich Mine. The values of tangential stresses for the indicated locations around the opening ($\eta = 0^{\circ}, 13^{\circ}, 90^{\circ}$) are given in Table 3.